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On The Role of Stepped Overlays to Increase Spillway Capacity of
Embankment Dams

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On the role of stepped overlays to increase spillway capacity of embankment dams

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ABSTRACT: The use of pre-cast concrete blocks or RCC stepped overlays for overtopping protection has proven to be cost effective and has gained acceptance, particularly in Russia and in the USA. The present paper includes a brief outline of particular project details and hydraulic design data of stepped spillways over embankment dams. Based on experimental data gathered on laboratory flumes and on a large-scale chute, representative of actual embankment dams, empirical models are proposed for estimating the flow properties down the chute slope. Recommendations for the hydraulic design of stepped chute overlays are also included.

1 INTRODUCTION

Although a considerable number of dams were built with overflow stepped spillways during the 19th and early 20th century (Chanson, 2000), this technology has greatly decreased until its rebirth in the 70s.

In recent years, a significant number of embankment dams have been considered unsafe due to inadequate spillway capacity and predicted overtopping during extreme flood events. Common structural remedial measures may include modifications on the dam (i.e., raising the top of dam to increase storage), increasing spillway capacity (e.g., by lowering the crest, increasing its width), construction of new spillways (emergency spillways, fuse plugs), and providing for safe overtopping. The latter solution has gained particular acceptance for existing small embankment dams, which constitutes the majority as regards recent ICOLD statistics of dam failures (Berga 1995). Vegetation, geotextile, riprap, gabions, reinforced concrete, pre-cast concrete blocks and RCC overlays are among the techniques developed to date for overtopping protection of embankment dams.

This paper discusses the use of both pre-cast concrete block systems and Roller Compacted Concrete (RCC) for protection of small embankment dams. Pre-cast blocks are most applicable in locations where use of large machinery is limited or where use of RCC would not be cost effective due to small quantities.

2 OVERTOPPING PROTECTION. EXAMPLES OF APPLICATION AND PROTOTYPE EXPERIENCE

2.1 *Pre-cast concrete tapered block system*

The design of pre-cast concrete tapered block system began to be introduced into hydrotechnical construction in the early 70s, in Russia, under the leadership of P. I. Gordienko (Moscow Institute of Technology). Gordienko performed numerous laboratory investigations of various designs of protective coverings of earth overflow dams and as a result proposed a design consisting of an in situ concrete overflow bulkhead on the crest and relatively gentle downstream slope protected by V-shaped reinforced-concrete slabs. Subsequent studies of Yuri Pravdivets (Moscow Institute of Technology) were aimed at eliminating the shortcomings of Gordienko's design (flatness of the profile, presence of an hydraulic jump along the slope, difficulties in protecting the side earth slopes in the hydraulic jump zone, various types of pre-cast re-vetments).

Outside Russia, significant laboratory and prototype or near prototype (i.e., at a scale representative of actual embankment dams) studies have been conducted in UK and in the USA. The laboratory tests conducted at Salford University, UK, have shown that failure of a stepped block system in unidirectional flow has never been witnessed (Baker 1992, 2000a). Further work developed in UK has led to the publication of a CIRIA Design Guide "Design of Step Block Spillways", where

topics such as geotechnical considerations, stilling arrangements, underdrain, minimum block size, spillway construction and post-construction have been covered (Baker 2000b).

The research program conducted in the USA by the US Bureau of Reclamation, the Electric Power Research Institute and Colorado State University (CSU) began with laboratory flume studies of various step shapes, and proceeded to test the best design in a scale representative of actual embankment dams. The tests showed that a properly design stepped overlay is inherently stable because of the flow on the step surfaces and the ability of the stepped overlay to relieve the uplift pressure. Guidelines for block shape and thickness, block system, filter and energy dissipation can be found in Frizell (1997).

Relvas (1997) developed a computer program for preliminary design of non-conventional spillways (including reinforced concrete channel, gabion type channel and pre-cast concrete blocks stepped channel). The analysis of three selected Portuguese dams has shown that using pre-cast concrete blocks to replace conventional works would decrease the cost of the spillway by an average of 60%. Pinheiro & Relvas (2000) present the most relevant results obtained for the cost estimates of the several non-conventional spillways analysed.

Custódio & Pinheiro (2000) developed a more detailed study of the use of stepped spillways made with pre-cast concrete blocks laid over the downstream slope of three small embankments that had been recently overtopped, as well as over a larger one, whose spillway capacity was found to be insufficient. They concluded that the use of that type of stepped spillway to complement the existing discharge capacity was a fairly economical solution.

Information on prototype installations of pre-cast concrete block system for overtopping protection of embankment dams is given in Table 1. Of the thirteen listed projects, five ranged in height from 7.0 to 8.0 m, five were between 11.0 and 13.0 m and two (excluding Dneiper special test chute) are large dams with heights of 20 m and 26 m, respectively. Design unit discharges are in the range 1.8 - 36.5 m²/s.

Of the installations listed in Table 1, Bolshevik, Klinbeldin, Maslovo and Sosnovski were built for state farm dams using standard reinforced concrete slabs providing a stepped configuration. The fairly flat slope of the channels (between 7 and 11 degrees with the horizontal) were determined by the characteristics of the embankment material (Bramley et al. 1989). Design unit discharges are in the range 3.0 to 3.3 m²/s.

Two of the pioneering prototypes constructed in the Moscow region have failed during the first years of operation, owing to not fulfilling the requirements imposed on the drainage-filter underlying the revetment, as stated by Pravdivets (1992). No indication is included, however, on the name and characteristics of the earth dams that failed. Another prototype failure, the Jelyevski dam in Ukraine, 1976 (it is possible that this refers to one of the installations previously addressed by Pravdivets 1992), was caused by loss of underdrain and embankment material through the holes in the blocks during first operation (Baker 2000b).

Despite these early unsuccessful designs, the step block protection has performed satisfactorily on several embankment dams in Russia, namely on the experimental earth dam constructed in the Magadan region and at Dneister cofferdam. The stepped spillway chute of the earth dam constructed in Magadan region has experienced ice and water discharge from early spring to late fall in an unregulated regime for 15 years. The wedge blocks used at the Dneister power station were installed on a 20 m wide section of the cofferdam, and successfully withstood several floods (with overtopping discharge intensities of up to 13 m²/s) and two ice-floes which occurred in 1978 and 1979.

Prototype tests have also been conducted on a special test chute at the Dneiper hydro plant, in Ukraine, 1976. The test section containing the blocks was 36.1 m long and 14.2 m wide with a longitudinal slope of 1V:6.5H (i.e. 8.8 degrees with the horizontal). The test channel was operated under controlled conditions for a total of 10 hours, at a maximum unit discharge of 60 m²/s. The majority of the blocks were displaced 2 to 3 cm from their original positions and in two localized areas the deformations reached 0.5 to 0.7 m (Bramley et al. 1989). Despite the considerable movements, the blocks remained in position and the results of these full scale tests showed that suitably-sized wedge blocks can withstand very large unit flow rates.

Relevant prototype or near prototype tests were more recently conducted outside of Russia, namely in the UK and in the USA, respectively. Brushes Clough (Greater Manchester, UK) was the first stepped block spillway constructed outside of Russia. The spillway was regularly monitored for the first two years of its life, as reported by Baker (1995, 2000b). The highest flow that occurred naturally during the monitoring period was 0.75 m³/s, while in the two surge test programs, flow rates ranging from 0.7 to 0.9 m³/s (1st test) and 2.02 m³/s (2nd test) were released. No block movement occurred.

Table 1 - Pre-cast concrete block overlays over embankment dams.

Dam	Ref.	Dam height (m)	Chute slope (deg.)	Chute width (m)	Type of blocks	Step height (m)	Design unit disch. (m ² /s)	Remarks
Bolshevik, Russia, 1980	BM, BA2	11.5	6.8 to 11.3	12	OS		3.3	Farm dam
Brushes Clough, UK, 1859 – 1993 (new overflow)	BA1	26.0*	18.4	2.0	WO	0.125	1.8	Full scale trapezoidal test channel; Q _{design} = 3.66 m ³ /s
Dneiper hydro plant, Ukraine, 1976	PB	37.0	8.8	14.2	WO	0.5	60**	Full scale tests; V ≤ 23 m/s
Dnesteir cofferdam	BA2	7.0	12.5	20	BW		13**	Withstood several floods; V _{max} = 8 m/s
Jelyevski, Ukraine	BA2	8.0	7.1	12			36.5	Dam failure
Jiangshe Wanan, China	BA2		11.3		BW	0.52		
Klinbeldin, Russia, 1976	BM, BA2	7.5	9.0	15	OS	0.2	3.0	Farm dam
Lukhovitsky dams, Russia, 1978, 1980 and 1981*	KP	11.0		12			3.0	3 earthen dams
		11.5		12			3.3	
		7.5		7.5			2.9	
Magadan region***	PR, BM, BA2	20.0	26.6	6			5.0	Farm dam. Frequent ice and water discharge
Maslovo, Russia, 1981	BM, BA2	7.5	8.8	7.5	OS		3.0	Farm dam
Sosnovski dam, Russia, 1978	PB, BA2	13.0	9.5	12	OS		3.3	Earth dam
Trans-Baikal region, Russia, 1986	MI	11.7	14.0	110	WO		19.1	Earth dam. Frequent overflows.
Wadi Sahalnawt, Oman, 1991	BA2		15.9	980		0.2	4.5**	Aquifer recharge dam
							5.1	

Note: OS - overlapping slabs; BW - Butt-jointed and wedge shaped; WO - Wedge shaped and overlapping.

* in Chanson (1994); ** maximum recorded unit discharge; *** it is possible that this refers to Kolyma dam (same installation). BA1 - Baker (1995); BA2 - Baker (2000a); BM - Bramley, May & Baker (1989); MI - Miller, Pravdivets & Salov (1987); KP - Krest'yaninov & Pravdivets (1986); PB - Pravdivets & Bramley (1989); PR - Pravdivets (1992).

Interesting key observations were drawn from the routine testing at Brushes Clough (Baker 1995, 2000b). The first one related to the behavior of the blocks under vandalism leading to the conclusion that for most small and medium sized spillways, the hydraulic factors are not critical for sizing the blocks. The main criterion is an adequate thickness of concrete to resist the impact of boulders and stones being dropped onto the steps. The second observation refers to the operational problems associated with the presence of stones and boulders on the channel. One of those operational problems is the disruption of the flow pattern due to a large

obstruction, causing jetting and splash which may prevent the formation of skimming flow and possibly de-stabilize the spillway lining. The other is due to the fact that smaller stones collect under the nappe in the lee of the step and could start to obstruct the holes through the block.

In USA, a near prototype facility located in Colorado State University (CSU) in Fort Collins, Colorado, has been used to investigate the hydraulic conditions on stepped surfaces (Frizell 1997, Frizell et al. 1994, 2000). The block shape was successfully tested for unit discharges up to 3 m²/s.

Table 2 - RCC stepped overlays over embankment dams in USA.

Dam	Ref.	Dam height (m)	Chute slope (deg.)	Step height (m)	Design unit disch. (m ² /s)	Max. overflow height (m)	Remarks
Ashton, 1991	MH	18.3	33.7	0.6	11.2	3.7	
Bishop Creek No. 2, 1989	MH	12.5	18.4	0.3	2.2	0.9	New emergency spillway
Boney Falls, 1989	MH	7.6		0.6	9.2	3.0	
Brownwood Country Club, 1984	MH	5.8	26.6	0.2	2.3	1.7	Overtopped 6 times since 1985, by less than 0.3 m
Butler Reservoir, 1992	MH	13.1	21.8	0.3	12.6	4.0	Steps hold at 1V:0.6H
Comanche Trail, 1988	MH	6.1	26.6	0.3	5.5	1.8	
Goose Lake, 1989	MH	10.7	45.0	0.5	0.8	0.7	
Goose Pasture, 1991	MH	19.8	18.4	0.3	8.7	3.0	
Harris Park No. 1, 1986	MH	5.5		0.3	8.3	3.0	
Horsethief, 1992	MH	19.8	26.6	0.3	1.6	1.3	
Kemmerer City, 1990	MH	9.4	21.8	0.3	2.2	1.1	
Kerrville, 1985	FR, MH	6.4	45	0.3	31.1	31.1	New RCC section; overtopped by 4.4 m (1985) and by 4.9 m (1989); b = 182 m
Lake Diversion, 1993	MH	25.9	20.0	0.2	29.0	6.2	New emergency spillway
Lake Lenape, 1991	MH	5.2	21.8	0.3		0.9	Steps compacted at 1V:0.6H
Lima, 1993	MH	16.5	26.6	0.6	5.6	2.8	Steps cut at 1V:1H
Meadowlark lake, 1992	MH	8.5	18.4	0.3	10.8	3.1	
North Potato Diversion, 1992	MH	10.7	11.3	0.6	31.2	6.1	New spillway
Philipsburg dam No. 3, 1992	MH	6.1	26.6	0.3	1.3	2.1	
Ringtown No. 5, 1991	FR, MH	18.3	20.0	0.3	5.1	2.1	Combined principal and emergency spillway. Frequently overtopped. Cut steps vertical.
Rosebud, 1993	MH	10.1	26.6	0.3	5.0	2.1	
Salado Creek, Site 10	RK	17.1	21.8		14.5		
Spring Creek, 1986	MH	16.2	23.1; 17.9	0.3	4.1	1.4	
Thompson Park, No. 3, 1990	MH	9.1	14.0	0.3	2.8	1.3	Overtopped 3 times by less than 0.03 m.
Umbarger, 1993	MH	12.2	18.4	0.3	19.8	5.3	Steps cut at 0.6H:1V
Upper Las Vegas Wash Retention	FR	18.3	20.0		21.4		
White Cloud, 1990	MH	4.6	21.8	0.2		0.5	
White Meadow Lake, 1991	MH	6.1	21.8	0.3		0.4	

FR - Frizell (1992); RK - Rice & Kadavy (1996); MH - McLean & Hansen (1993).

2.2 RCC stepped overlays

Since the first use of RCC to provide increased safety during overtopping for Ocoee No. 2 dam, a 9.1 m high rock-filled timber crib dam constructed in USA, 1913, the RCC protection technique has gained acceptance for overtopping protection of embankment dams, particularly in the USA. By the end of 1998, the number of US embankment dams rehabilitated using RCC totaled 58 (Hansen & Bass 1999), and a total of 72 are currently being rehabilitated or planned for rehabilitation in the near future (Bass 2000). Other projects have already been constructed worldwide, namely in South America (McLean & Hansen 1993).

A summary of projects where RCC stepped overlays have been used for overtopping protection of embankment dams is shown in Table 2. Of the twenty seven listed projects, twelve ranged in height from 4.6 to 9.4 m, six from 10.1 to 12.5 m and nine are large dams with heights from 16.2 to 25.9 m. Design unit discharges are in the range 0.8 - 31.2 m²/s.

Although most of the RCC stepped overlays for overtopping protection have not yet been subjected to large flows, close to the design value, some have experienced significant overflows (e.g., Ocoee 2 dam), while others have passed low flows (e.g., Brownwood Country Club dam, Thompson Park No. 3 dam and Ringtown N. 5 dam). A review of the performance of such projects is included in McLean & Hansen (1993).

Kerrville Ponding dam, a 6.4 m high, 182 m long concrete-capped clay embankment was built in 1980 (Hansen & Reinardt 1991). A portion of the dam in the spillway area was washed out in late 1984 when a flood overtopped the embankment by 3 m. After evaluating several alternatives, a new RCC section was constructed immediately downstream of the damaged embankment area. The dam has experienced two significant overtoppings: shortly after completion of the RCC section (1985), when heavy rains caused the dam to be overtopped by as much as 4.4 m (1-50 year event), and in July 87, the dam was subject to a 100-year event, which overtopped it by a maximum of 4.9 m. No noticeable erosion or other distress resulted (Hansen & Reinardt 1991).

The RCC unstepped overlay of the detention dam at North Fork, designed to operate as service spillway, has experienced frequent overflows (McLean & Hansen 1993). The entire flow of the river passed over the RCC section for eleven months. For the first six months, river flows passed over the structure at velocities estimated to be 4.6 to 6.1 m/s. During this period the reservoir filled with sediment and rocks to the spillway crest elevation. For the following five months water-borne

debris passed over the structure and gravel rocks up to 0.3 m in diameter were observed to passing over the RCC surface at high velocities. But it was only in 1982, after another eruption of the volcano, that additional debris clogged the reservoir which resulted in the embankment overtopping and breach in two places.

Ringtown No. 5 dam, completed in 1991, has experienced several low head flows with satisfactory performance and no noticeable problems. The Brownwood Country Club dam, the first earth embankment in USA rehabilitated using RCC, has been overtopped at least six times since 1985 with a maximum height over the RCC protected embankment estimated at 0.3 m. The Thompson Park dam has experienced three overtoppings of small height, about 2.5 cm. The latter two RCC stepped overlays performed without of any kind of distress being noted (McLean & Hansen 1993).

3 HYDRAULICS OF SKIMMING FLOW OVER STEPPED OVERLAYS ON EMBANKMENT DAMS

3.1 Presentation

In stepped spillways, distinct flow regimes can be found for identical chute geometry (i.e., step configuration), namely the nappe flow regime at low unit discharges, the mixed or transition flow regime at intermediate flow rates and the skimming flow at larger unit discharges (e.g., Essery & Horner 1978, Ohtsu & Yasuda 1997).

In stepped chute overlays for overtopping protection of embankment dams, the nappe and mixed flow regimes occurs in general for unit discharges considerably lower than that corresponding to the design value. The characterization of the skimming flow properties down the stepped chute is of particular relevance for the hydraulic design of the chute sidewalls as well as the energy dissipator at the spillway toe.

Similarly to high-velocity flows on conventional spillways, skimming flow down stepped spillways can be divided into a number of distinct regions. In the non-aerated flow region close to the spillway crest, the boundary layer grows from the spillway floor. Outside the boundary layer the water surface is initially smooth and glassy, but it becomes contorted, upstream of the inception of air entrainment. This contorted surface is responsible for the transport of air between the irregular waves, as shown in Matos et al. (1999). At the point of inception, where the boundary layer reaches the free-surface, entrainment of air by the multitude of vortices in the turbulent flow commences. Downstream of the start of air entrainment an upper layer

containing a mixture of air and water develops with increasing depth. In skimming flows, the rate of growth of the above layer is significant in a short region close to the point of inception. Downstream of that region, a trend of slight increase of the air concentration with the distance is noticeable. Far downstream from the point of inception the flow becomes quasi-uniform and for a given discharge, the flow properties such as the mean air concentration, equivalent clear water depth and mean water velocity will not vary along the spillway. The parameters of air concentration, clear water depth, inception point depth, Froude number and slope distances are used in the hydraulic design of stepped chutes.

3.2 Definitions

The local air concentration C is defined as the time averaged value of the volume of air per unit volume. The equivalent clear water depth is defined as

$$d = \int_0^{Y_{90}} (1-C) dy \quad (1)$$

where y is measured perpendicular to the spillway surface and Y_{90} is the depth where the local air concentration is 90%. A depth averaged mean air concentration for the flowing fluid can then be defined from

$$d = (1-C_{\text{mean}}) Y_{90} \quad (2)$$

The average water velocity U_w is defined as

$$U_w = \frac{q_w}{d} \quad (3)$$

3.3 Flow properties at the point of inception

On stepped chutes, the position of the inception of air entrainment is mainly a function of the flow discharge, step geometry and chute geometry. Chanson (1994) re-analysed the flow properties at the point of inception of model experiments and the following formulae have been developed for chute slopes ranging from 26.6 degrees (1V:2H) to 53.1 degrees (1V:0.75H)

$$\frac{L_i}{k} = 9.719 (\sin \alpha)^{0.0796} F^{0.713} \quad (4)$$

$$\frac{d_i}{k} = \frac{0.4034}{(\sin \alpha)^{0.04}} F^{0.592} \quad (5)$$

where L_i is the distance from the start of growth of the boundary layer, d_i the flow depth at the point of inception and F a roughness Froude number

defined as $F = q_w / \sqrt{g \sin \alpha k^3}$, where $k = h \cos \alpha$ (h is the step height and α the chute slope).

The application the above formulae is expected to overestimate slightly L_i as well as d_i (Matos et al. 1999, 2000a), because the data used to obtain Eq. (4) were based on the visual observations of the apparition of "white waters", and also because Eq. (5) was mostly based on the sidewall bulked flow depths at the point of inception. However, Eqs. (4) and (5) have been adopted herein because they were developed for chute slopes ranging from 26.6 to 53.1 degrees, hence applicable to the CSU chute, where the data for the location of the point of inception and the respective flow depth were not available.

3.4 Mean air concentration down the stepped chute overlay

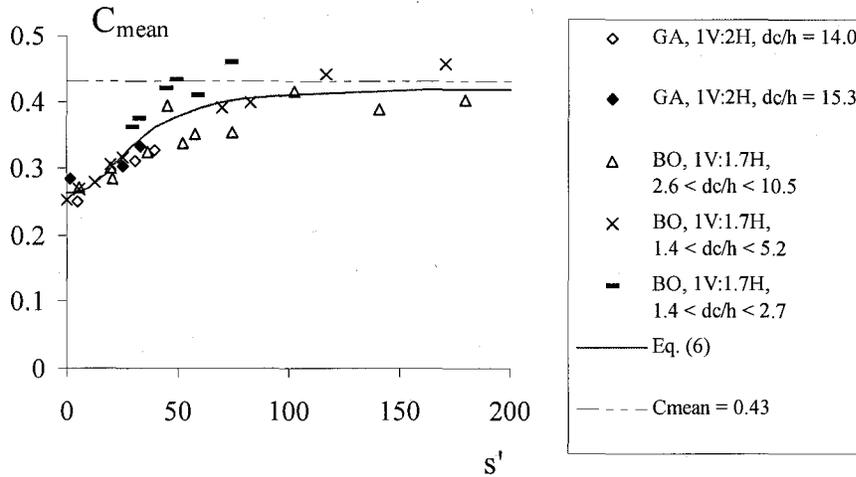
Figure 1 shows the growth of the mean air concentration in skimming flow down stepped chute overlays, as a function of the normalized distance s' ($s' = (L - L_i)/d_i$), where L is the distance measured along the chute. In this figure, a more complete set of experimental data gathered by Boes at the 30 degrees sloping chute assembled at ETH-Zürich, has been included.

The present re-analysis of data of Gaston (1995) along with the more complete set of data by Boes (2000), obtained for values of s' up to 180, fits fairly well to the equation

$$C_{\text{mean}} = 0.262 + \frac{0.158}{\left[1 + (0.031s')^{-2.389}\right]}, s' > 0 \quad (6)$$

The free-surface deformation transporting entrapped air along with the flow is responsible for the large values of the mean air concentration close to the inception point, as illustrated in Figure 1 and shown in Eq. (6) ($C_{\text{mean}} \sim 0.26$ for $s' \sim 0$).

Two markedly distinct regions can be observed in Figure 1: a first region where the mean air concentration (C_{mean}) increases very rapidly (s' lower than about 50), and a subsequent region where a gradual increase of C_{mean} is noticeable. The considerably large scatter of the data for s' close to 50 might be due to the different values of the dimensionless parameter dc/h (ratio of the critical depth over the step height). Near the downstream end of the chute and for large values of s' (i.e., $s' > 100$), C_{mean} approaches the equilibrium value for self-aerated flow on conventional chutes of identical slope, which is 0.43 for a 30 degrees sloping chute, according to Matos (1999) (after Wood 1991, Hager 1991, Chanson 1993). This result is in accordance with



Experimental data: GA – Gaston (1995); BO – Boes (2000); s' – normalized distance given by: $s' = (L-L_i)/d_i$; d_c – critical depth.

Figure 1. Mean air concentration down the stepped chute (after Frizell et al., 2000).

the reasoning by Frizell et al. (2000) that uniform flow was not reach for $s' < 100$ (based on the data by Gaston 1995 along with the data by Boes & Hager 1998).

3.5 Equivalent clear water depth and characteristic depth down the stepped chute overlay

Similarly to the mean air concentration profile, the experimental data gathered at CSU and at ETH-Zürich stepped chutes were re-analysed herein to obtain the dimensionless equivalent clear water depth (d/d_i) in function of the normalized distance s' , as shown in Figure 2. Even though the scatter of the data is large for s' lower than 50, the trend of the data is reasonably represented by the equation

$$\frac{d}{d_i} = 0.642 + 0.105 e^{(-0.011 s')} \quad (7)$$

At the point of inception ($s' = 0$), Eq. (7) gives $d/d_i = 0.75$, because the flow depth estimated by visual observation is larger than the equivalent clear water depth.

From Figure 2 it can be seen that the decrease of the equivalent clear water depth is more pronounced for s' lower than about 100. For larger values of s' , the decrease of d/d_i is negligible (e.g., the relative difference on the equivalent clear water depth for $s' = 100$ and 200 is less than 5%). Hence, the quasi-uniform equivalent clear water depth is

approximately equal to 0.65 times the flow depth at the inception point, the latter being estimated by Eq. (5).

Figure 3 compares the experimental values of the dimensionless characteristic depth (Y_{90}/d_i) with those estimated by the application of Eqs. (2), (6) and (7). In the quasi-uniform flow region, the characteristic flow depth approaches 1.2 times the flow depth at the point of inception.

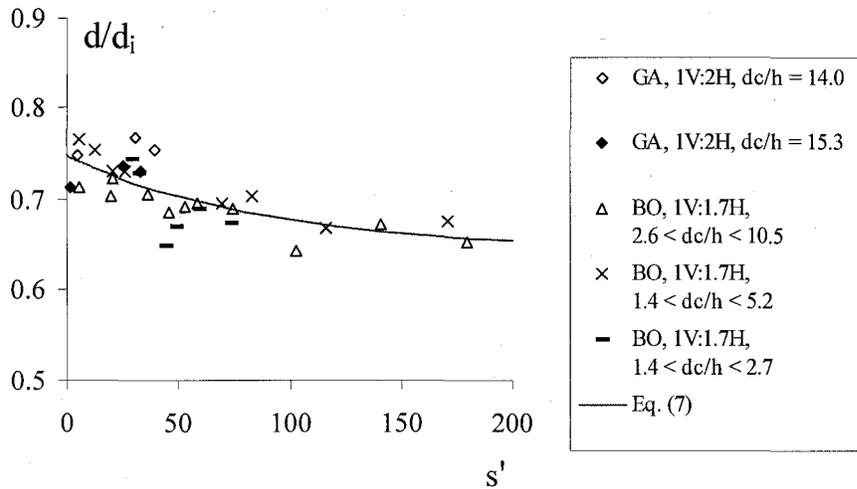
4 HYDRAULIC DESIGN CONSIDERATIONS

4.1 Flow properties at the point of inception

The location of the inception point and the respective flow depth can be estimated by Eqs. (4) and (5) (Chanson, 1994). Eq. (6) allows the estimation of the mean air concentration close to the point of inception ($s' \sim 0$).

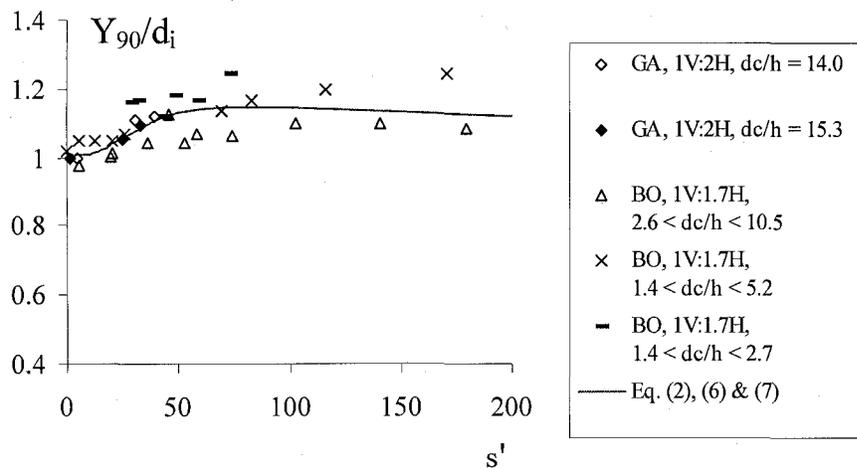
4.2 Flow properties downstream of the point of inception

The mean air concentration and the equivalent clear water depth are given by Eqs. (6), (7), along with Eqs. (4) and (5). Eq. (2), along with the former equations, can be used to obtain the characteristic flow depth Y_{90} . Wood's (1985) turbulent diffusion model of the air bubbles within the air-water mixture or the novel advection-diffusion model proposed by Chanson (1997) can be used to predict the air concentration distribution



Experimental data: GA – Gaston (1995); BO – Boes (2000); s' – normalized distance given by: $s' = (L-L_i)/d_i$; dc – critical depth.

Figure 2. Dimensionless equivalent clear water depth down the stepped chute.



Experimental data: GA – Gaston (1995); BO – Boes (2000); s' – normalized distance given by: $s' = (L-L_i)/d_i$; dc – critical depth.

Figure 3. Dimensionless characteristic depth down the stepped chute.

and, in particular, the air concentration close to the pseudo-bottom, as shown by Matos & Frizell (1997) and by Chanson et al. (2000), respectively.

4.3 Training wall height

The characteristic depth Y_{90} may be used in the design of the training walls. Although it

contributes little to the total discharge, the spray projected from the air-water wavy interface may extend well above Y_{90} . According to the findings of Boes (in Boes & Minor 2000), the mixture flow depth Y_{95} is about 12% larger than Y_{90} , whereas Y_{99} is approximately 40% larger than Y_{90} (Y_{95} and Y_{99} are the depths where the local air concentration is 95 and 99%, respectively). An additional safety factor of 1.5 (i.e. sidewall height equals $1.5 \cdot Y_{90}$),

as recommended by Boes & Minor (2000) for emergency spillways on embankment dams prone to erosion, is judged adequate.

4.4 Residual energy

The equivalent clear water depth, obtained from Eq. (7), along with Eq. (5), can be used to estimate the specific energy at the spillway toe. A value of about 1.2 may be used for the kinetic energy correction coefficient on stepped spillways over RCC dams, as proposed by Matos (2000) and by Boes & Minor (2000).

4.5 Potential for cavitation damage

Based on the study of the potential for cavitation damage at the point of inception on stepped spillways typical for RCC dams, Matos et al. (2000b) have concluded that the cavitation index is expected to be larger than the incipient cavitation index for unit discharges up to 20 to 30 m²/s, which corresponds to the mean velocities at the inception point between 17 and 23 m/s. Significant dissimilar conclusions are not expected for stepped chute overlays on the downstream slope of embankment dams.

Once it has been determined that cavitation will not occur upstream from or at the inception point, the increased air concentration near the pseudo-bottom downstream from the inception point will assure safety against cavitation damage at any point further downstream on the stepped slope of an embankment dam. It is interesting to note that cavitation damage was not reported in the prototype tests conducted at Dneiper special test chute under 35 m head, in which the unit discharges attained 60 m²/s (mean velocities up to 23 m/s).

5 CONCLUSIONS

The use of stepped overlays to increase the spillway capacity of embankment dams is a very attractive solution, which is becoming more widespread. The performance of most pre-cast concrete block systems and of the RCC stepped overlays in service has been satisfactory.

New empirical models were proposed for estimating the main flow properties down stepped chute overlays, namely the mean air concentration, equivalent clear water depth and characteristic flow depth. Recommendations for the hydraulic design of stepped chute overlays were also included.

Basic design criteria is already available for spillways made with pre-cast concrete blocks, allowing a reasonably accurate design. However, further research is required, in order to clarify

some aspects like flow patterns in trapezoidal channels and seepage effects for lower discharges. With regard to the design criteria of RCC overlays for embankment protection, further studies are still needed (McLean & Hansen 1993, Berga 1995), namely on the pressure field on the step surfaces and its effect on the footing due to the cracks in the RCC, as well as on the hydraulic erosion of poorly compacted areas of the exposed RCC.

Should continuing research be carried out in a near prototype facility, simulating not only the stepped overlay, but also the embankment underneath, it will be possible to obtain information and criteria which may give more confidence to design engineers.

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